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GEOMETRICALLY NONLINEAR ANALYSIS OF STEEL STORAGE RACKS SUBMITTED TO EARTHQUAKE LOADING

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Steel storage racks are light and flexible structures. When submitted to earthquake loading, they can exhibit very large transverse displacements and are thus prone to significant consequences of second-order geometrical effects. In the context of the drafting of European recommendations for the design of steel pallet racks for their seismic resistance, this paper presents a parameter study comparing the various methods commonly used in practice for analyzing the seismic structural behavior of racks (i.e. "modal response spectrum analysis" and "lateral force method analysis") as well as the different ways to account for geometrically nonlinear effects in these conventional methods of analysis in the case of structures designed for low ductility.

Keywords: Steel storage racks; earthquake analysis; second-order effects.

1. Introduction

Static steel pallet racks are used for the storage of various types of goods in the areas of retail warehouse stores and other facilities, possibly accessible to the public. Storage racks are composed of specifically designed cold-formed steel elements permitting an easy installation and later reconfiguration, consistent with the merchandising needs of a warehouse retail store.

The classical configuration of pallets is approximately 1 sq m of plan areas and the maximum load weight is about 10−15 kN. Storage racks bays are usually 1.0−1.1 m deep and 1.8−2.7 m wide and can accommodate two or three of these pallets. The overall height of pallet rack structural frames, such as usually found in retail warehouse stores, ranges from 5 to 6 m. Racking systems can nevertheless reach much

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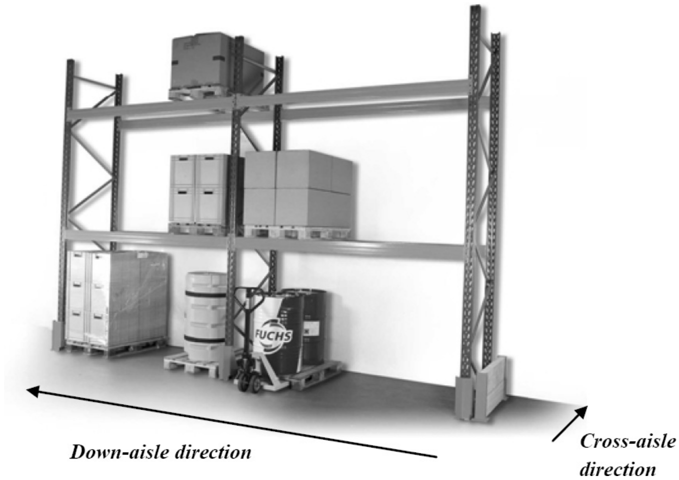

Fig. 1. Classical configuration of static steel pallet racks.

more considerable heights up to 12−15 m and even more in industrial facilities. Proprietary moment connection frames are traditionally used as structural system in the down-aisle (i.e. longitudinal) direction, while braced frames are typical for the cross-aisle (i.e. transverse) direction (Fig. 1). If required, braced frames are also used in down-aisle direction for overall stability reasons. Despite their lightness, racking systems carry very high live load, by far higher than the dead load, contrary to what usually happens in civil engineering structures.

The prediction of the structural behavior of pallet racks is far from obvious because affected by the particular geometry of their structural components. Indeed their members are made of high slenderness thin-walled elements hence prone to global, local, and, for uprights, distortional buckling problems. Moreover, beam-to-upright and base-plate connections exhibit a strongly nonlinear behavior. Due to these peculiarities, specific modeling and design rules are required for these non-traditional steel structures. Reference can, therefore, hardly be made to classical structural design recommendations and standards.

2. Design Methodologies

The most recent design standards[1−4] for steel storage racks under static loading recommend to perform a combined numerical-experimental approach in which the design structural analysis is supported by specific tests to evaluate the performance of the key components (members and joints).

The design is even more complicated for storage racks installed in seismic areas, as shown by some recent studies dealing for instance with performance-based design of pallet-type steel storage racks,[5] shake table testing with or without seismic isolators,[6,7] cyclic behavior of beam-to-upright joints[8] or dynamic interaction between

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the rack structure and the stored goods.[9] In particular, it is worth mentioning a research project funded by the European Commission from 2004 to 2007 that covered a wide amount of topics[10]: cyclic behavior of connections, sliding of pallets, assessment of behavior factors, etc.

Conventional seismic design of structures implies the verification of two types of limit states: (i) ultimate limit states (ULS), associated with the structure collapse or with other forms of structural failure that may endanger the safety of people and (ii) damage limitation states (DLS) associated with damage beyond which the specified requirements are no longer met. In general, ULS are based on resistance verification whereas DLS are related to displacements limitation. According to a decision of the FEM 10.2.08[11] drafting committee (European Recommendations for the design of racks under seismic conditions), DLS have been replaced by *a posteriori* assessment of damage, meaning that DLS no longer need to be considered in the design. In fact, after a seismic event, the damages caused by the earthquake to the structural elements must be assessed before continuing the usage of the rack. In practice, it results in no sway displacements limitation under seismic action. Moreover, since rack structures are highly flexible, ULS requirements (stability and resistance) can be fulfilled even if sway displacements are significant. However, this requires of course that the second-order geometrical effects be duly taken into account in the global structural analysis. Currently, static steel pallet racks can be considered as part of the most flexible structures constructed nowadays, in particular for what concerns the down-aisle behavior.

Under static conditions, the designer may refer to EN15512[2] for instance, where it is stated that a second-order analysis must be performed or, alternatively, that a first-order analysis may be used, provided internal forces be amplified by an appropriate coefficient (in a similar way to the *amplified sway moment method* of the Eurocode 3).

The situation is somehow different under seismic action because of two main reasons:

(1) Seismic action on a given structure depends on the dynamic properties of this structure and in particular on its first natural period of vibration. The geometrical second order effects tend to decrease the average lateral stiffness of the structure, leading to increased values of the period. In the usual range of periods for racks (see Sec. 4), the spectral acceleration — and consequently the seismic action on structure — decreases when the period increases. Therefore, the second-order geometrical effects are likely to reduce the action applied by the earthquake to the rack.
(2) When assessing the safety level of structures submitted to static loading, it is implicitly supposed that all loads (i.e. vertical and horizontal) increase in a proportional way. In other words, increasing horizontal loads are applied on a structure whose lateral stiffness is progressively degraded due to the increasing compressive loads acting vertically. On the contrary, in the event of an

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earthquake, the seismic safety is assessed by considering first the vertical gravity loads and second the horizontal loads representative of the seismic action. The flexibility of the structure is thus higher than the one of the nonloaded structures but is considered constant during the whole seismic event.

These two statements evidence the fact that the second-order geometrical effects should be treated in a different way for earthquake loading than for normal static loading.

In practice, regarding rack structures, two main references are available:

(1) Very recently, the American Rack Manufacturer Institute (RMI) has issued an update of its document "*Specification for the Design, Testing, and Utilization of Industrial Steel Storage Racks*".[1] These recommendations obviously require a due account for the second-order effects and, regarding the earthquake situation, make reference to the FEMA 460[12] report. In this report, it is suggested to carry out a first-order analysis then to account for P-Delta effects by amplifying the results. The amplification factor depends on the rotational stiffness of the beam-to-upright and base plate connections. This approach is, therefore, implicitly dedicated to unbraced frame structures.

(2) In Europe, the main document is the draft version of FEM 10.2.08 "*Recommendations for the design of static steel pallet racks under seismic conditions*" issued by the European Racking Federation. The recommendations of this document are mainly based upon the philosophy of EN 1998-1 (Eurocode 8)[13] although the peculiar dynamic behavior of racking structures and their stored unit loads is included. Given that the rules take root in the Eurocode 8, the management of the P-Delta effects is based on the so-called θ parameter (*interstory drift sensitivity coefficient*) defined according to Eq. (1) and Fig. 2.

$$\theta = \frac{P_{\text{tot}} d_r}{V_{\text{tot}} h} \tag{1}$$

where P_{tot} is the total gravity load at and above the considered story; d_r is the design interstory drift evaluated as the difference of the average lateral displacement at the top and bottom of the considered story. This displacement is

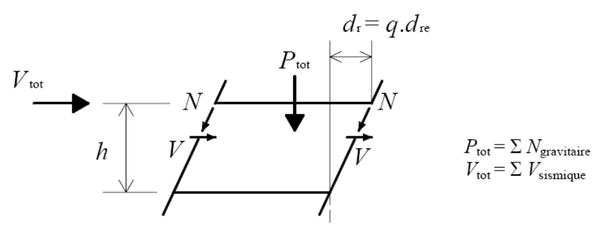

Fig. 2. Parameters used to calculate the interstory drift sensitivity coefficient θ.

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obtained using a first-order elastic analysis and multiplied by the behavior factor q considered in the design; V_{tot} is the total seismic story shear; and h is the interstory height.

It is reminded that the behavior factor q accounts for energy dissipation in the structure through a reduction of the elastic spectrum (see also Sec. 4). According to FEM 10.2.08, this factor ranges between 1.5 and 4 for rack structures, according to the choice of nondissipative or dissipative designs and to the structural typology.

The parameter θ is thus characteristic of each story and should be managed in a differentiated way for each level. In practice, a conservative approach consisting in considering a single value of θ for the whole structure (i.e. the highest obtained from the whole set of stories) is often followed.

According to the Eurocode 8, the consequences on the structural analysis corresponding to a given range of θ can be summarized as in Table 1(a). However, in view of the usual values of θ obtained for rack structures (see Sec. 4), Table 1(a) has been qualified as highly severe by rack designers and producers. In particular, the absolute upper limit of 0.30 was proven to be a very strong design criterion implying significant longitudinal bracings to fulfill this requirement only. Therefore, the limit values have been adapted in FEM 10.2.08 leading to Table 1(b). Additionally, if the structure is designed to behave elastically under seismic action — following Eurocode 8 principles, it means that the structure is designed according to DCL concept (ductility class low), resulting in a behavior factor q ranging between 1.5 and 2.0, FEM 10.2.08 allows to use the "amplified first-order method" up to $\theta = 0.50$.

Table 1(a). Impact of θ values on the structural analysis according to Eurocode 8.

θ value	Consequences
$\theta \leq 0.10$	Second-order effects do not need to be taken into account
$0.10 < \theta \leq 0.20$	Second-order effects may approximately be taken into account by multiplying the relevant seismic action by a factor equal to $1/(1-\theta)$
$0.20 < \theta \leq 0.30$	Design action effects must be obtained by a nonlinear method of analysis ("pushover" or nonlinear time-history analysis)
$\theta > 0.30$	Not allowed

Table 1(b). Impact of θ values on the structural analysis according to FEM 10.2.08.

θ value	Consequences
$\theta \leq 0.10$	Second-order effects do not need to be taken into account
$0.10 < \theta \leq 0.30$	Second-order effects may approximately be taken into account by multiplying the relevant seismic action obtained from a first-order analysis by a factor equal to $1/(1-\theta)$
$0.30 < \theta \leq 0.50$	Design action effects must be obtained by a nonlinear static method of analysis ("pushover" analysis)
$\theta > 0.50$	Time-history analysis including large displacements and nonlinear behavior of material and connections is required

3. Objectives

The study presented in this paper takes place in the above general context, the aim of which is assessing the prescriptions proposed in FEM 10.2.08. More precisely, the paper deals with down-aisle frames and follows three definite objectives:

(1) Evaluate the sensitivity parameter θ for rack structures such as designed in practice;
(2) Evaluate the actual impact of the second-order effects in seismic context for such flexible structures; and
(3) Compare different methods of global analysis and, in particular, assess the accuracy of the simplified approach proposed by FEM 10.2.08 for intermediate values of θ.

The results presented herein constitute the first part of the study and deal with analyses in the elastic domain. The geometrical second-order effects (P-Delta) are considered independently of any material nonlinearities or plastic dissipation. These results have been elaborated in the frame of a master thesis realized at the University of Liège.[14] The interaction between the sensitivity factor θ and the behavior factor q is currently under study and will be published later on.

Section 4 presents the elaboration of a structural database comprising nine structures without longitudinal bracing (unbraced frame structures) and nine structures with longitudinal bracing designed according to EN 15512 and FEM 10.2.08. For each structure, the sensitivity parameter θ and the fundamental period of vibration are evaluated. Section 5 describes the different methods of elastic analysis that will be considered for comparison purpose and Sec. 6 presents the results obtained when applying the different methods on the 18 case studies.

4. Structural Database

In the present work, only down-aisle behavior is considered. Regarding the cross-aisle direction, structural systems of pallet racks are commonly made of Z, D or X trusses and these systems are known to be less sensitive to second-order effects.

A set of 18 rack structures is designed under the following geometrical assumptions (see also Fig. 3):

— The pallet mass equals 0.66 tons;
— The beams are 2.7 m length (three pallets per bay); the total length equals the number of bays (Nb_{Bays}) times 2.7 m; and
— The cell is 1.5 m height; the total height equals the number of stories (Nb_{Stories}) times 1.5 m.

The seismic action is defined according to the Eurocode 8.[12] The fundamental period of flexible structures and of rack structure in particular is rather high. Type 1 spectrum is thus chosen since it is characterized by a less decreasing branch in the

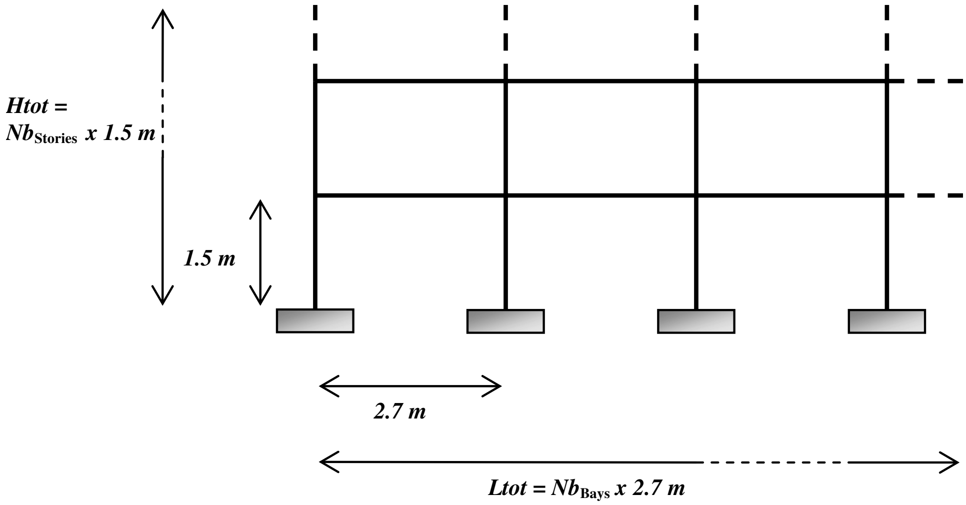

Fig. 3. Global geometry of the structures.

range of long periods. Typical shape of type 1 spectrum providing the pseudo-acceleration of the structure versus its period is shown in Fig. 4.

The elaboration of the present case studies is based on a moderate seismicity level. The ground acceleration a_g is taken equal to $1.25\,\text{m/s}^2$. The soil type is considered as average (type C) leading to the following values of the control parameters of the spectrum:

— The soil coefficient S equals 1.15.
— The control periods T_B, T_C, and T_D, respectively, equal 0.2 s, 0.6 s, and 2.0 s.

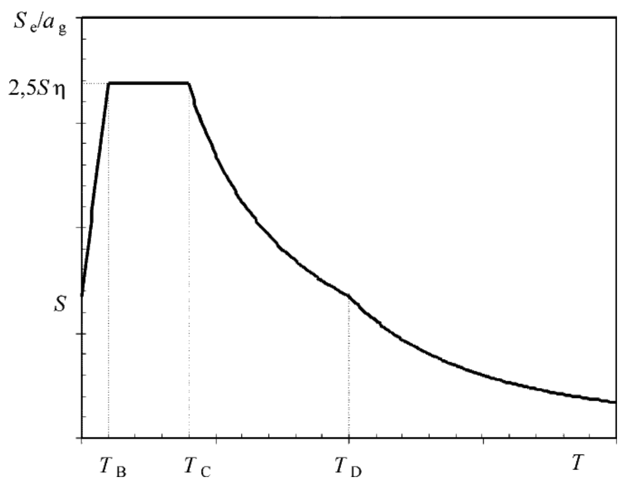

Fig. 4. Recommended shape of the elastic response spectrum according to Eurocode 8.

According to FEM 10.2.08, it is allowed to account for limited energy dissipation in down-aisle frames submitted to earthquake provided that:

— For moment frame structures, the members contributing to the seismic resistance of the structure by working in compression and/or bending are made of class 1–3 profiles.
— For braced structures, the bracings are made of class 1–3 profiles and composed of diagonals acting in tension and compression (X bracings).

The above conditions are assumed to be verified in the design examples. Under these conditions, the energy dissipation, although moderate, may be accounted for by the use of a behavior factor q equal to 2.0 reducing the acceleration obtained from the elastic spectrum. As a counterpart, the displacements obtained from the first-order analysis will have to be multiplied by 2 to calculate the value of the sensitivity parameter θ (see Eq. (1)).

The set of case studies has been defined to target a wide range of sensitivity coefficients. For this purpose, different combinations of bays and stories were considered. By way of consistency, the profiles have been chosen in the catalog of the Stow International company with geometrical and mechanical properties duly validated according to the EN 15512 prescriptions. Upright sections are classical rack open sections with dimensions ranging from 85×65 mm to 120×92 mm according to the configuration. Racks beams are characterized by a rectangular hollow section realized from two C profiles. Their dimensions are ranging from 100×50 mm to 120×50 mm.

The beam-to-upright and upright-base connections semi-rigid properties have also been characterized according to the same standard, including a due account for the influence of the vertical compression on the stiffness and the resistance of the upright-base connections. Last, the properties of longitudinal X-bracings have also been determined similarly. However, none of these properties is explicitly provided in the present paper because of confidentiality reasons.

For structures without longitudinal bracings, various combinations of profiles for the beams and uprights were also considered for given combinations of bays and stories, leading to complementary structural variants.

In this design stage, the seismic structural effects are determined using an amplified lateral force method (see Sec. 5 for the methodology), whatever the value of the sensitivity coefficient and considering the initial lateral stiffness for the estimation of the fundamental period.

Two load combinations are considered for the design checks, i.e. the weighted gravity load and the nominal gravity load combined with the seismic action. For both combinations, the following verifications are made:

— upright stability;
— beam deflection;

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— connection resistance (beam-to-upright and upright-base); and
— bracing resistance when relevant.

Tables 2(a) and 2(b) present the designed structures respectively without and with longitudinal bracings. The tables include the number of bays and stories, the fundamental period and the sensitivity coefficient θ. They also provide the fundamental period obtained for the same structure if the reduction of lateral stiffness caused by the nominal compressive load in the uprights is accounted for.

From this list, it can be observed that:

— The sensitivity coefficient θ for rack structures with bracings ranges between 0.1 and 0.3. According to Table 1(b), it is thus always necessary to account for the second-order effects.
— The sensitivity coefficient θ for rack structures without bracings can be by far higher than 0.3. During the development of the database, it has even been possible to design a "4 story–5 bay" structures with a θ coefficient equal to 0.79 and fulfilling all design criteria (NB: this structure has, however, not been kept for the forthcoming comparison, since considered as excessively flexible). This shows that

Table 2(a). Case studies, structures without longitudinal bracings.

Number of stories	Number of bays	Sensitivity coefficient θ	Natural period (initial stiffness)	Natural period (reduced stiffness)
2	3	0.20	0.91 s	0.96 s
2	3	0.23	0.96 s	1.00 s
2	3	0.25	1.01 s	1.07 s
2	3	0.28	1.07 s	1.14 s
3	3	0.29	1.25 s	1.33 s
3	3	0.33	1.31 s	1.41 s
3	3	0.34	1.39 s	1.50 s
3	3	0.36	1.38 s	1.50 s
3	3	0.40	1.46 s	1.60 s

Table 2(b). Case studies, structures with longitudinal bracings.

Number of stories	Number of bays	Sensitivity coefficient θ	Natural period (initial stiffness)	Natural period (reduced stiffness)
7	5	0.10	0.87 s	0.89 s
8	6	0.13	1.06 s	1.09 s
8	8	0.15	1.20 s	1.23 s
8	10	0.18	1.31 s	1.36 s
8	12	0.20	1.41 s	1.46 s
8	14	0.22	1.50 s	1.56 s
9	14	0.25	1.67 s	1.75 s
10	14	0.27	1.86 s	1.95 s
11	13	0.29	1.99 s	2.10 s

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the absolute limit on θ (0.3) proposed by the Eurocode 8 can govern the design and lead to the use of longitudinal bracings significantly increasing the structural cost.

— Even in a situation of moderate seismicity and with a behavior factor of 2, it is hardly possible to design a rack structure with more than three levels without longitudinal bracings.
— The fundamental period of rack structures range between 1.0 and 2.0 s.
— The reduction of the lateral stiffness due to compression of the uprights can lead to a significant elongation of the fundamental period (up to 10%), even for braced structures. Indeed, braced rack structures can actually be considered as hybrid structures (in opposition to pure trusses) since the horizontal forces are partly transmitted by the bracing system and partly by a frame effect. The loss of bending stiffness of the uprights due to compression consequently influences the global structural stiffness through its "frame" component. Therefore, the increase of fundamental period also appears in the case of braced structures. This type of hybrid resisting system is chosen by rack designers to reduce the cost by limiting the bracings to the very necessary minimum.

5. Methods of Analysis

As previously announced in the objectives, this paper exclusively deals with structures designed for low ductility ($q = 1.5$ to 2.0). It is thus assumed that their behavior is elastic under seismic action.

Three classes of methods can be considered for the seismic analysis of linear elastic structures:

(1) According to most design codes, the reference method for determining the seismic effect shall be the "modal response spectrum analysis" (MRSA) using a linear elastic model of the structure. This method provides the dynamic response of the structure by using appropriate combinations of its natural modes of vibrations.
(2) For structures the response of which is not significantly affected by contributions from modes of vibration higher than the fundamental mode in each principal direction, it is allowed to substitute MRSA by the use of equivalent static forces ("lateral force method of analysis" — LFMA). The conditions under which LFMA can be used are explicitly given in most seismic codes and in the Eurocode 8 in particular.
(3) It is of course also possible to perform a dynamic time-history analysis of the system (THA). In this case, the procedure consists in defining a set of ground motion time-histories (accelerograms) representative of the seismic action at a given location. These accelerograms can be either natural (provided that adequate databases are available) or artificial. In the present paper, only artificial ground motions are considered and a set of seven time-histories compatible with the reference spectrum of Fig. 4 is generated with the software GOSCA.[15]

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In the case of linear elastic structures and if the three methods are used under all the required assumptions (in particular for LFMA), the average of the structural responses obtained using THA based on all the set of considered time-histories should yield the same results as MRSA, while LFMA yields a reasonably safe approximation of the response and thus slightly higher values of the displacements and internal forces.

Coming now to geometrically nonlinear analysis, the three above methods should appropriately be modified to account for the second-order effects.

5.1. *MRSA*

As described in Sec. 3, the main second-order effect in rack structures is a loss of bending stiffness of the uprights due to compression induced by the weight of stored goods. It means that the earthquake action should be applied on a structure with reduced stiffness. A first possibility would thus be to perform a two-step procedure: first, analyze the structure submitted to the gravity loads and deduce the compression level in the uprights, on the basis of which an update of their transverse stiffness can be carried out; second, perform an MRSA on the structure with reduced stiffness. This procedure is, however, not considered herein because of the following reason: MRSA is based on a superposition of the normalized modal shapes factored by coefficients that depend on the modal mass (and thus indirectly on the modal shape) and on the spectral acceleration (and thus on the period). It can be shown that, when modifying the uprights stiffness, the periods associated to the vibration modes vary but the modal shapes remain more or less unchanged. Therefore, the only consequence of the first step of the method is to elongate the period, accounting thus only for the beneficial effects of second-order.

An alternative solution is to follow the proposal of the Eurocode 8 and of FEM 10.2.08, i.e. first perform a linear elastic MRSA and then amplify the results in terms of internal forces by $1/(1-\theta)$, where θ is defined by Eq. (1). It is worth pointing that for structures designed for low ductility, the behavior factor q can be taken equal to 1.5−2.0 to account for some energy dissipation. However, the source of this dissipation is not explicit plastic dissipation that would necessarily require an amplification of the displacements d_r. Consequently, in the next comparisons, two hypotheses are considered:

(1) The MRSA results are amplified by $1/(1-\theta)$ where d_r is obtained from the linear elastic analysis and multiplied by q;
(2) The MRSA results are amplified by $1/(1-\theta)$ where d_r is directly obtained from the linear elastic analysis without the use of the factor q.

5.2. *LFMA*

For this type of analysis, the restriction expressed above regarding the use of reduced stiffness does not hold. Indeed, earthquake actions are modeled by equivalent forces

Table 3. Summary of the methods of analysis.

Reference	Description
MRSA-a	Modal response spectral analysis amplified by $1/(1-\theta)$ with θ based on amplified displacements
MRSA-b	Modal response spectral analysis amplified by $1/(1-\theta)$ with θ based on nonamplified displacements
LFMA-1-a	Lateral force method analysis assuming a linear elastic behavior of the structure amplified by $1/(1-\theta)$ with θ based on amplified displacements
LFMA-1-b	Lateral force method analysis assuming a linear elastic behavior of the structure amplified by $1/(1-\theta)$ with θ based on nonamplified displacements
LFMA-2	Lateral force method analysis assuming a linear elastic behavior of the structure with stiffness reduced by compression due to gravity loads
LFMA-3	Lateral force method analysis assuming a nonlinear elastic behavior of the structure
NLTHA-M	Nonlinear time-history analysis considering the maximum response obtained from the seven accelerograms
NLTHA-m	Nonlinear time-history analysis considering the mean response obtained from the seven accelerograms

applied on a structure for which the reduction of stiffness has a direct influence on the calculated effects. The following different hypotheses are considered in the next paragraphs:

(1) The results obtained by conventional linear elastic analysis are factored by $1/(1-\theta)$. As for MRSA, two assumptions are made for the calculation of θ (d_r amplified or not by q).
(2) The internal forces are calculated using a two-step procedure: the structure is first submitted to gravity loads and a linear elastic analysis is performed; the uprights stiffness is then updated using classical stability functions in which the calculated level of compression is considered. Equivalent lateral forces are finally linearly applied on the stiffness-reduced structure to derive the seismic effects.
(3) A fully nonlinear elastic analysis is carried out on the structure considering the actions of both gravity and lateral seismic forces.

5.3. *THA*

In order to serve as reference, some of the presently considered structures are also studied using nonlinear time-history dynamic analysis with the set of seven accelerograms previously mentioned.

Table 3 summarizes the approaches that are further compared in Sec. 6. All the analyses are performed with the nonlinear finite element software FineLg.[16]

6. Results and Comparisons

Comparison between the different approaches is achieved based on the significant internal forces taking place in the structure. No comparison is performed in terms of displacement since the Damage Limit States are not to be considered for storage structures.

For structures without longitudinal bracings, the monitored forces are the bending moments at the base of the most loaded upright M_{Base} and at the most loaded beam-to-upright connection $M_{Beam-to-upright}$ (in practice corresponding to the beams of the first story). For braced structures, the monitored forces are the bending moment at the base of the most loaded upright M_{Base} and the tension force in the most loaded diagonal bracing N_{Brace} (in practice corresponding to the diagonals of the first story).

Since the reference method is MRSA, the results are always expressed as the ratio of the internal force obtained with a given analysis method to the value of the same internal force obtained with nonamplified MRSA. The results plotted in the different graphs are thus the values of the amplification factor of the corresponding internal force.

For sake of clarity and before any comparison be made among the methods accounting for the geometrical nonlinearities, a first comparison between linear elastic MRSA and linear elastic LFMA without any kind of second-order amplification is provided. The bending moment ratios at the base of the most loaded upright are compared in Fig. 5 for the 18 structures of the database. They are plotted in function of the sensitivity coefficient θ.

Before commenting on Fig. 5, the conditions of application of LFMA are usefully reminded. According to Eurocode 8, LFMA may be applied if

$$T_1 \leq \begin{cases} 4T_C \\ 2.0\,\mathrm{s} \end{cases} \tag{2}$$

where T_1 is the fundamental period of vibration and T_C is the second control period of the design spectrum (see Fig. 4). For type 1 spectrum, T_C is equal to 0.6 s and the governing limit is thus $T_1 \leq 2.0\,\mathrm{s}$. According to Tables 2(a) and 2(b), LFMA can be applied to all structures of the database. It must, however, be underlined that high structures have a fundamental period of vibration very close to the limit.

The total seismic shear force is determined using the following expression

$$F_b = S_d(T_1)m\lambda \tag{3}$$

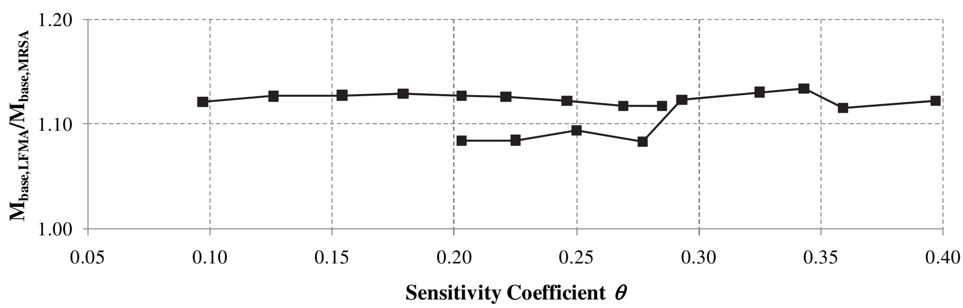


Fig. 5. Comparison between linear elastic MRSA and LFMA for the bending moment at the upright base.

where $S_d(T_1)$ is the ordinate of the design spectrum at period T_1, m is the total mass of the structure, and λ is a correction factor depending in the fundamental period T_1. If $T_1 \geq 2T_C$ (i.e. 1.2 s), the correction factor is equal to 1.0, which is the case for most structures of the database.

Figure 5 clearly shows that, under the above assumptions, LFMA constitutes a conservative simplification of MRSA with a safety margin ranging from 8% to 13% according to the actual modal mass of the structure.

The second comparison is carried out between nonamplified MRSA and NLTHA-m, M. As already recalled previously, MRSA and linear-THA theoretically yield the same results in terms of structural response. Therefore, if MRSA is considered as the reference, the results obtained with NLTHA should allow the evaluation of the effects of geometrical nonlinearities (favorable and unfavorable). The comparison is provided in Table 4 for a subset of the database. For NLTHA-M, the values of the internal forces given in Table 4(a) are the very maximum values of the forces recorded during the seven step-by-step analyses carried out (i.e. one analysis for each accelerogram), while for NLTHA-m, the values are the average of the maximum values obtained from each of the seven analyses. According to Eurocode 8, NLTHA-m values are the values to be considered for the design or assessment of structures when NLTHA are used.

Table 4(b) shows that the consequences of second-order effects are rather moderate (within a range of −6% to +20%), even for sensitivity factors up to 0.4 and even if the

Table 4(a). Comparison between nonamplified MRSA and NLTHA (absolute values).

Not braced	M_{Base} [kNm]			$M_{Beam-to-Upright}$ [kNm]		
θ	MRSA	NLTHA-m	NLTHA-M	MRSA	NLTHA-m	NLTHA-M
0.20	2.43	2.36	2.68	2.41	2.27	2.63
0.29	3.38	3.46	3.69	2.60	2.57	2.81
0.40	3.07	3.23	3.45	2.18	2.09	2.46
Braced	M_{Base} [kNm]			N_{Brace} [kN]		
θ	MRSA	NLTHA-m	NLTHA-M	MRSA	NLTHA-m	NLTHA-M
0.20	3.42	3.61	4.00	64.2	70.06	77.27

Table 4(b). Comparison between nonamplified MRSA and NLTHA (percentage).

Not braced	M_{Base} [%]			$M_{Beam-to-Upright}$ [%]		
θ	MRSA	NLTHA-m	NLTHA-M	MRSA	NLTHA-m	NLTHA-M
0.20	—	−3	+10	—	−6	+9
0.29	—	+2	+9	—	−1	+8
0.40	—	+5	+12	—	−4	+13
Braced	M_{Base} [%]			N_{Brace} [%]		
θ	MRSA	NLTHA-m	NLTHA-M	MRSA	NLTHA-m	NLTHA-M
0.20	—	+6	+17	—	+9	+20

extreme value obtained from the full set of ground motions is considered. This is due to the compensating effects of period elongation and increase of structural flexibility.

It can also be underlined that, based on the comparison of Table 4 and Fig. 5 and if only the mean value of the time-history results is of concern, the nonlinear

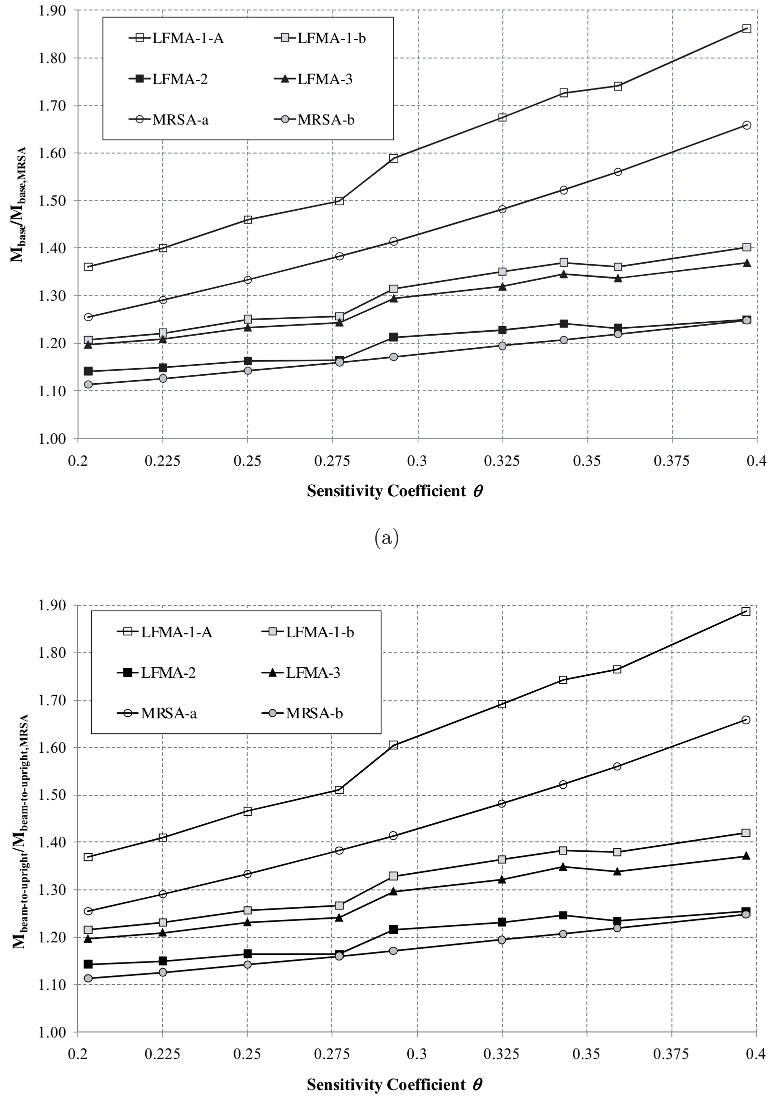
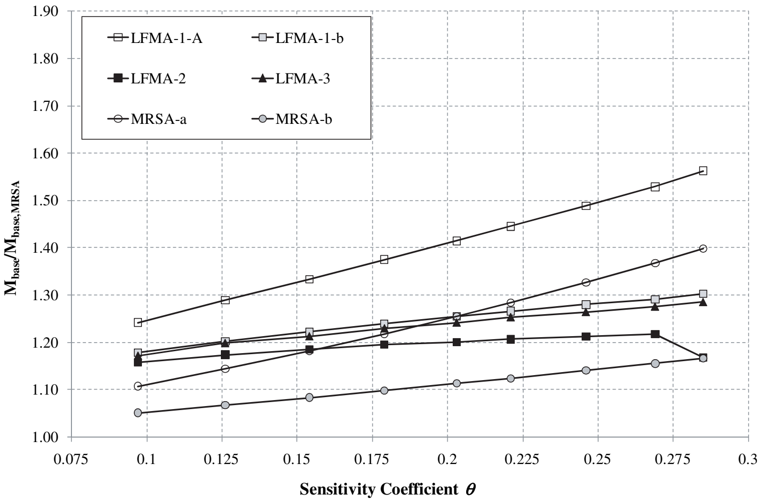
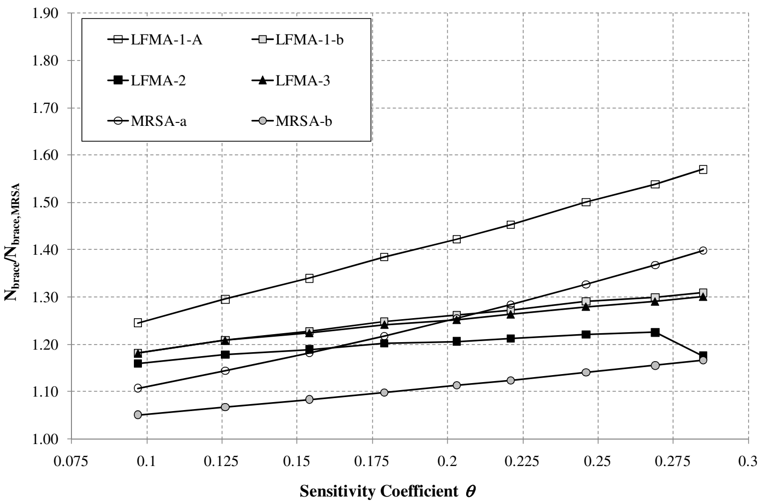


Fig. 6. (a) Comparison among the analysis methods for the bending moment at the upright base (structures without bracings), (b) comparison among the analysis methods for the bending moment at beam-to-upright connection (structures without bracings), (c) comparison among the analysis methods for the bending moment at the upright base (braced structures) and (d) comparison among the analysis methods for the tension forces in the diagonal bracings (braced structures).



(c)



(d)

Fig. 6. (*Continued*)

amplification with respect to linear MRSA results (+9% in the worst situation) is smaller than the safety margin provided by the correction factor λ in LFMA.

Figures 6(a) to 6(d) present a comparison among the different variants of LFMA and MRSA listed in Table 3 and accounting in a way or another for geometrically nonlinear effects. For all cases, the reference is the corresponding internal force obtained with linear elastic MRSA. The results are plotted versus the sensitivity coefficient.

From these figures, it can be observed that:

- The scatter of the estimated amplifications is fairly high for a given sensitivity coefficient. For instance, it ranges from 25% to 85% for $\theta = 0.4$. It is also worth recalling that for the same situation ($\theta = 0.4$), NLTHA yields an amplification of only 13%.
- Except for low values of θ, the amplification curves appear to be sorted in the following ascending order: MRSA-b, LFMA-2, LFMA-3, LFMA-1-b, MRSA-a and LFMA-1-a.
- LFMA-3 and LFMA-1-b yield very close results.
- Except for LFMA-1-a and MRSA-a, all the other curves exhibit a very similar slope, proportional to $1/(1 - \theta)$, and it can be reasonably supposed that the curves slopes extend beyond $\theta = 0.4$.
- When compared to NLTHA mean or even maximum values, all methods are safe, except for MRSA-b that leads to some unsafe results when compared to maximum values.
- There is no evidence that conclusions should be differentiated for braced and unbraced structural systems.

Bearing in mind that the database considered for the study is relatively limited, a number of conclusions can be drawn regarding the application of the proposed methods accounting for the second-order effects for use in the context of storage racks:

- Due to the rather long fundamental period of rack structures, the possible use of "lateral force method analysis" must be checked carefully. Even if the method does not seem to provide unsafe results, it should be underlined that the internal forces considered in the present paper for the purpose of comparison are not the most liable to suffer from higher modes influence, those are definitely known to considerably more affect the upper levels. This particular point should certainly deserve complementary investigations.
- For the considered range of periods and sensitivity coefficients, all the considered methods are conservative with respect to the mean values obtained with the dynamic analysis using the full set of ground motions. The safety level is, however, fairly scattered and the use of the amplified first-order analysis for structures having a high-sensitivity coefficient is seen as significantly over-conservative, in particular for unbraced structures. Therefore, it is thought that the amplification could be performed without considering the behavior factor q in the calculation of the sensitivity coefficient θ. However, if the safety level is chosen so as to be conservative with respect to the most challenging ground motion, the "amplified modal response spectrum analysis" should always include the behavior factor q within the calculations.

7. Conclusions

The structural database elaborated for the specific objectives of the study presented in this paper aimed at validating the recommendations of FEM 10.2.08 for steel pallet rack structures regarding the analysis methods. The structures experienced in the present research study are highly flexible structures so as evidenced by their long fundamental periods (up to 2 s) and the high values of sensitivity coefficients to second-order effects (θ from 0.1 to 0.4).

Different methods of analysis accounting in various ways for geometrically nonlinear effects are described and their results in terms of internal forces ratios are then duly compared.

A first comment drawn using the results of time-history nonlinear analysis (although performed on a limited amount of configurations) is that the amplification of the internal forces induced by second-order effects is relatively limited (up to 20%), partly thank to the favorable influence of the period elongation.

Another major comment that should be underlined is that all conventional methods (i.e. "modal response spectrum analysis" and "lateral force method analysis") are conservative with respect to nonlinear dynamic analysis whatever the type of structural system (braced or unbraced), partly because the behavior factor q is included in the calculation of the sensitivity coefficient. The level of conservatism is nevertheless highly varying from an approach to another.

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